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Geotechnical Prediction and its Influence on the Chosen Method of Excavation

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SYNOPSIS Difficult ground was predicted for the 6,3 km long Grauholz Tunnel. The geology comprises complex soft-ground above and below the groundwater and rock conditions. The examples described led to the choice of a mixshield machine to excavate the 11,6 m diameter tunnel. This machine can operate in either closed (slurry) or open (TBM) mode. Experiences with this installation are discussed.

1. INTRODUCTION

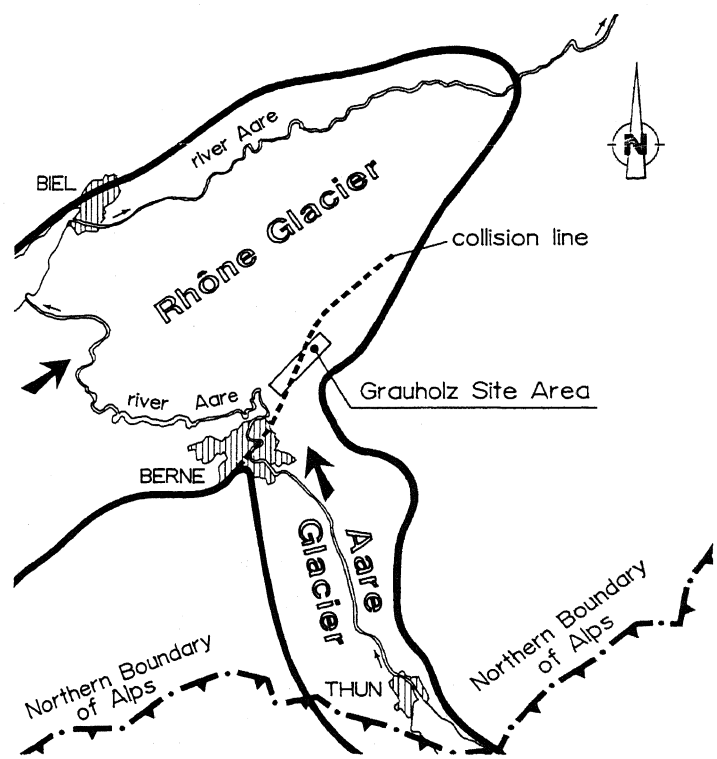
The Grauholz Tunnel is a double track railway gallery of a total length of appr. 6,3 km. It has been under construction since mid-1988 and will be, according to schedule, ready for operation by May 1995. At each end of the tunnel is a section which was constructed by cut and cover. From the geotechnical point of view the most challenging part was, however, the 5,5 km long central section being mined since January 1989. The breakthrough is expected for end of April 1993.

2. GEOLOGICAL SITUATION

The Grauholz Tunnel lies north of Berne, capital of Switzerland (Fig. 1). This means geologically it is situated within the forelands of the Alps, about 30 km north of the strongly folded and faulted, northern alpidic boundary. The bedrock formation within the Grauholz site area comprises therefore the so called Unfolded Molasse of Miocene age, a sedimentary sequence which is also practically unfaulted. Sandstone, marl and mundstone are the prevailing lithology. The Molasse is a soft rock due to a low degree of diagenetic consolidation.

Quartenary fluvial and glacial erosion moulded the Molasse rock surface on a pattern which does not correspond to the actual ground morphology. A valley in the rock surface can be covered by a hill at surface level. The prediction of thegeological underground conditions by surface mapping becomes even more difficult because the erosion and sedimentation processes which alternately took place followed a scheme which was (and still is) more or less unknown in detail. Figure 1 depicts the conditions assumed for the end of the last glaciation of the forelands (i.e. the last Würm ice maximum appr. 12000 ys. B.C.). There was a collision of two main glaciers, the Aare Glacier advancing from the Bernese Alps in the south and the much larger Rhône Glacier which came from the west, the present area of Lake Geneva. This took place in the

FIGURE 1 Geological situation durring the end of the last ice-age



Grauholz area and resulted in the deposition of different types of moraines and of fluvioglacial fans and river sediments. Water damming structures and small lakes were formed which were consecutively filled with fine grained varved layers and coarser delta sediments, which were possibly overflown and overconsolidated by the forwardpushing glaciers at a later stage.

3. GEOTECHNICAL SITE EVALUATION

In the view of this geological history of the site area, the evaluation and prediction of the geotechnical conditions along the tunnel axis had to take into account the possibility of an extremely complex geologic and hydrogeologic underground structure. Different field investigation campaigns were necessary to determine the conditions to be expected. Geophysical methods (refraction seismic, VLF) and direct exploration by borings and large dia. wells were used. Different insitu-tests were performed in the boreholes (SPT, Menard pressuremeter tests, shear vane tests, constant and variable head tests, pumping tests under athmospheric and vacuum conditions). Piezometer and porewater pressure cells were installed to monitor different perched and phreatic groundwater tables.

One alignment of the tunnel favoured by the Authority and the Engineers for more than three years had to be given up due to unfavorable hydrogeological conditions. The finally chosen axis of project leads through soft-ground and rock in nearly equal proportions. The schematic longitudinal section (Fig. 2) shows however that starting the tunnel drive to the east a first 740 m long section (13%) with soft-ground is followed by the rock section which has a total length of 2'750 m (49%). The western soft-ground section with an overburden maximum of 75 m is about 2'060 m long (38%).

The evaluation of field and laboratory research work resulted into the differentiation of totally nine geotechnical units of which at least three were dominant in the eastern soft-ground section whereas the western section had to be driven over 1'600 m in one unit and only in the westernmost part additional units had to be expected.

4. CHARACTERIZATION OF GEOTECHNICAL UNITS

The identification of geotechnical units, finally leading to the implementation of the geotechnical model, is necessary and important but was difficult due to the transient nature of glacial deposits. From a total of nine geotechnical units identified along the Grauholz Tunnel the afore-mentioned four units most frequently encountered in the soft-ground section and the Molasse rock are geotechnically characterized in Table 1.

TABLE 1. Geotechnical properties of geotechnical units; the abbrevations d - i are discussed in the text; only the dominant composition is indicated under USCS; γ = unit weight, PI = Plasticity index, ϕ' = effective angle of friction, c' = effective cohesion, σ = uniaxial strength, k = coefficient of permeability; the given figures are mean values gained in the laboratory, except the k-value which is based on insitu tests

Geot. Unit	USCS	PI %	γ kN/m^3	ϕ' °	c' kPa	$k \cdot 10^{-4}$ m/s
d	ML,SM	5.4	21.0	35.2	1.3	0.5
e	CL	10.2	21.4	19	420	<0.01
g	GM	4.8	22.8	42.8	–	0.2
h	GP	–	–	43.3	–	3.2
i	Molas. rock	–	22.7–25.1	σ=9.0–22.1MPa		≤0.06

All these units were overconsolidated through at least 200 m thick glacial load during the last ice-age. Therefore, also the mainly non-cohesive units g+h have a high density. The properties of unit i vary considerably due to the different lithologies alternating within dm- to m-rhythm (i.e. mudstone, siltstone, and sandstone).

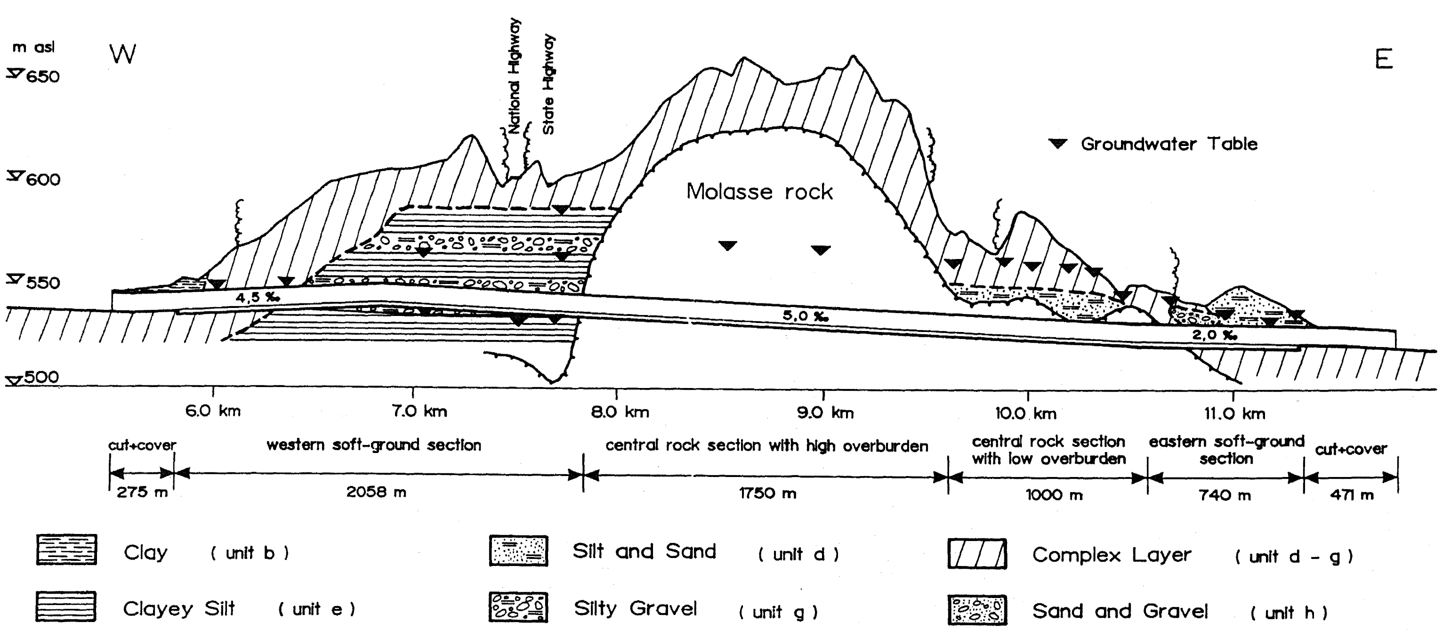


FIGURE 2 Schematic longitudinal section of the Grauholz Tunnel

The soil units d-h were classified as

d - delta sediment: stratified or crossbedded alternation of silt and sand beds

e - glacial lake sediment: clayey silt with mm-thick coarse silt to fine sand layers (varves), occasionally with erratic blocks and boulders

g - moraine: non-stratified silty to sandy gravel with stones, boulders and erratic blocks possibly up to onehalf of the tunnel diameter

h - fluvioglacial river sediment: sand and gravel

5. HYDROGEOLOGICAL CONDITIONS

Soft-ground conditions unaffected by groundwater are an exception at least in Switzerland. Within the Grauholz site the design groundwater table in the eastern soft-ground section is about 16 m and in the central rock section about 36 m above tunnel floor. Perched water at different locations and altitudes exists within the western section whereas the level of the tunnel could be fixed in such way that the phreatic water table in the western section would be mainly below the floor and only in the westernmost part up to 10 m above the floor.

The permeability of the different geotechnical units varies considerably over orders of magnitude.

6. PRELIMINARY GEOTECHNICAL CONCLUSIONS

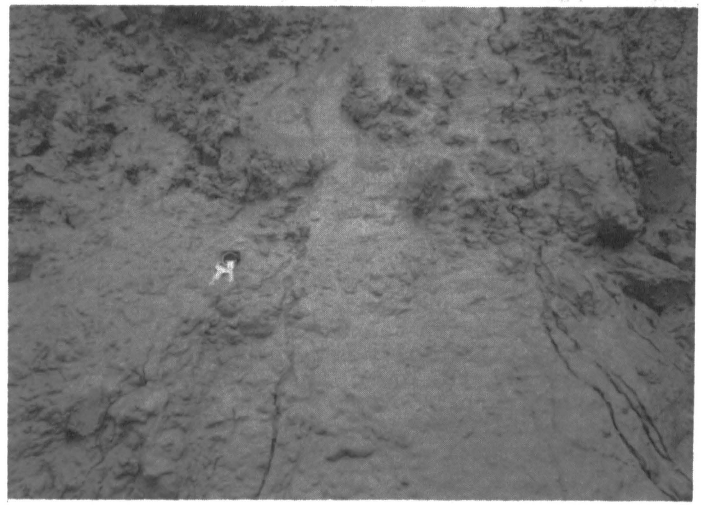
The available tunneling experience obtained in this type of ground revealed several geotechnical problems to be solved. The most important ones were:

- tunneling alternately in soft-ground and rock formation
- hard boulders and erratic blocks of great dimensions
- sand and gravel layers with low or no cohesion
- loss of cohesion of medium to coarse grained silt layers through water flow
- adhesive properties of clay layers to excavation tools
- muck problems with mudstones
- groundwater pressure up to 3,6 bars above floor

With regard to the predicted soil conditions and the wide radius required for the tunnel, the Engineers had planned to use a conventional shield drive. Ground support was to be provided by precast concrete segments. Tunneling in soft-ground under groundwater by using open face shield means that dewatering is necessary, for instance through wells drilled from the surface, as it was planned.

However, this project had to cope with a number of special problems. Three examples are given below:

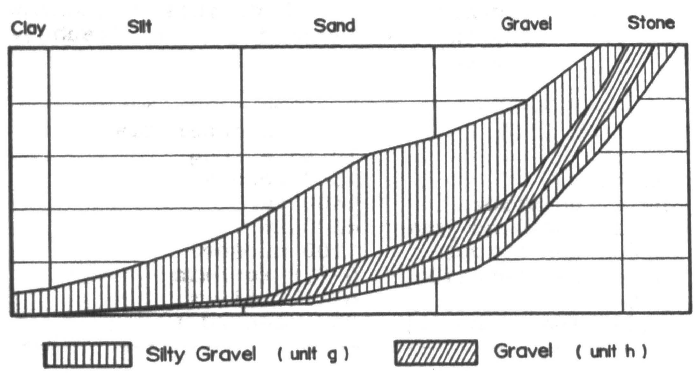
FIGURE 3 Flowing silt and sand (geotechnical unit d) because of insufficient dewatering



The geotechnical unit d behaves very unstably when insufficiently dewatered. Figure 3 shows flowing silty sand of unit d in parts of the starting pit where wellpoints came late into action. Open face tunneling under such conditions is not possible. The dewatering scheme for the underground section in unit d consisted therefore of two curtains of deep vacuum wells along both sides of the tunnel. The spacing of the wells was determined to be 4 m based on a water lowering trial. A further row of wells was planned along the middle axis of the tunnel because the irregular stratification and the associated variations of permeability of unit d could cause a very limited reach of the individual well. 250 vacuum wells of 6" diameter and up to 40 m depths were therefore included in the tender to prepare the drive of the tunnel section in unit d which is about 410 m long.

The second example considers the unit h. As can be derived from the grain size distribution envelope (Fig. 4) this uncohesive layer exhibits a "running" type of behaviour when under stress for instance through the forward pushing forces of shield blade. In order to avoid a front collapse grout stabilization of the underground either from the tunnel face or the surface were planned prior to the tunneling of the section in unit h.

FIGURE 4 Grain size distribution envelopes of the geotechnical units g+h



The last example deals with mixed face conditions predicted within a depression of the rock surface where three quarters of the face is formed by rock and the remaining upper part is covered by unit d+e beds. The groundwater pressure at rock surface is 2,6 bars. Lowering the pressure by conventional means nearly to zero would have been practically impossible although necessary with respect to the unit d - properties (see above). An expensive sealing and stabilization of the roof over a distance of about 80 m was designed involving concrete jet columns driven in advance from the tunnel face in 15 m steps.

These three examples make clear that the project as submitted in the tender documents had included difficult extra tasks to be solved. Failures were at least partially very probable. Moreover, the necessity to lower the groundwater table along the tunnel over a distance of about 1'650 m would have made a great impact on the landscape as well as on the groundwater itself.

7. CHOSEN METHOD OF UNDERGROUND EXCAVATION

In 1986/7 positive tunneling experience existed already using closed shields to retain the ground and water in the face by means of pressurised slurry or earth. However, this experience was only related to fine to medium grained softground and diameters up to appr. 7 m. The other alternatives of face support by compressed air or freezing the ground were judged negatively with respect to the Grauholz conditions.

The submitted offers did not only refer to the Engineer's project. After their detailed evaluation considering more geotechnical and environmental items than economical conditions a proposal offering a mixshield machine was finally deemed appropriate to excavate and construct the tunnel. This mixshield machine can operate in open (TBM) or closed (slurry) mode.

The main technical data of the machine are given in Table 2. In view of the mixed face conditions the machine has 67 roller disc cutters for dealing with erratic blocks and Molasse rock with a further 50 picks mounted behind the cutters for the excavation of softground. A crusher is installed in the center of the cutterhead to break boulders up to 1 m in diameter. The shield machine is followed by a 220 m long equipment train including the bentonite separation plant. The muck is discharged by continous belt conveyor.

TABLE 2. Technical data of mixshield machine chosen for tunneling the Grauholz Tunnel

Diameter	11,65 m outer dia.
Length	10,67 m long
Weight	1300 tonnes
Installed power	1400 kW
Total installed power	4500 kW
Cutterhead torque	1100 mtonnes
Cutterhead rotation	0-2,8 rev/min
Cutters	67 17" discs
Cutter load capacity	23 tonnes/disc

Ground support is provided by six precast concrete segments and a key segment. To provide watertightness each ring is sealed with EPDM compression gaskets and the annulus between the completed ring and the excavated diameter is filled with mortar grout.

8. SOME PRACTICAL EXPERIENCES AND CONCLUSIONS

Shield drive in slurry mode means that the slurry has to be replaced at least once a week by compressed air in order to carry out maintenance of the cutterhead installations. The formation of a complete slurry cake at the tunnel face is then of great importance and becomes even more essential considering the wide radius of the Grauholz bore and the great variation of permeability associated with the soft-ground layers described.

When working in compressed air the rule of thumb is often applied by which the depth of cover should be as thick as one diameter of the tunnel. The soil in the face should preferably consist of low permeable soil to keep the water out as well as to hold the air in. At the tunnel chainage km 10.920 (see Fig. 2) the overburden is only 9 m thick and the well permeable unit h was to excavate. The face collapsed at the beginning of a maintenance halt just after compressed air had overtaken the face support. The cutterhead jammed by the soil tumbled down and a crater of one tunnel diameter marked the buried machine at surface level. This collapse which caused a hold up of 4 months was probably due to incomplete sealing of the face by the slurry cake.

FIGURE 5 Hole formed at ground surface, three months later the tunnel was excavated at a depth of 50 m below surface

The importance of the "correct" adjustment of the slurry properties became once more evident when excavating unit g in the western soft-ground section. Although a felloe was installed at the cutterhead in order to reduce the collapse potential, material inflow from the tunnel roof into the slurry filled excavation chamber must have occurred during excavation of the tunnel at km 7,840 (Fig. 2). It remained undected till three months later a chimney had formed fortunately 20 m away from a state highway (Fig.5).

These experiences indicate that a certain amount of research work is deemed still necessary both to define the properties of bentonite slurry and its use in soil as given e.g. in Figure 4 and to invent measures to monitor permanently the amount of material beeing excavated.

One especially positive experience was that erratic blocks broadly discussed before starting the underground excavation were not a problem. They were obviously reduced to small pieces by the disc cutters before tumbling out of the face small enough for the crusher to do the rest of the break up work.

Great difficulties had to be settled separating the slurry from the excavated soil when fine grained layers such as the geotechnical unit e or mudstone (unit i) were dominant in the tunnel face. Special machines had to be installed and tested to improve the effectivness of the separation plant. Cement stabilization and vertical drains were finally necessary to dump appropriately the clayey material in prepared compartments.

Despite these and other problems the courageous decision to undertake the adventure of this new excavation method in difficult and variing ground was rewarded in regard to economy and ecology. The tunnel drive achieved an average advance of 8,9 m per working day or 6,5 m including the standstill time through exceptional events. The open face shield was offered exceeding the costs of the mixshield tunneling by 25% because two installations would have been necessary to complete the tunnel in the same time as it was done by the mixshield machine. Since the applied method did not affect the groundwater the communal drinkingwater wells adjacent to the tunnel could remain under operation during tunneling activities.